MILLS

Comparative Designs

for a Highway Bridge

Civil Engineering

B. S.

1912

# COMPARATIVE DESIGNS FOR A HIGHWAY BRIDGE

BY

GUY G MILLS

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THESIS

FOR THE

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IN

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This is to certify that the thesis of GUY G MILLS entitled COMPARATIVE DESIGNS FOR A HIGHWAY BRIDGE was prepared under my personal supervision; and I recommend that it be approved as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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Professor of Civil Engineering.

# COMPARATIVE DESIGNS FOR A HIGHWAY BRIDGE

Introduction

For a number of years the author has used the roads and bridges in the vicinity of Palestine, in Crawford Co., Illinois. Among the bridges which it has seemed wise should be immediately replaced by more substantial structures is one crossing the La Motte creek, about one mile from the point at which it empties into the Wabash river. A personal interest in this proposed improvement and a desire to make a general study of Highway Bridge Design led the author to take as a thesis subject a study of comparative designs for a new bridge at this particular site. The material thus submitted includes comparative designs and estimates for four different types of bridge structure:-

1. A single through steel truss span.
2. Two steel pony truss spans.
3. Two reinforced concrete girder spans.
4. A reinforced concrete arch.

Owing to the fact that the elevations of the bridge site are but little higher than the mean water level in the river, the principal flow in the creek at this point is due to back water from the river. The indications are that the maximum high water mark at the site is only about one foot below the top of the grade at the west end of the present bridge. Assuming the elevation of the bridge floor at the west end as datum a topographic survey was made and the notes plotted as shown in

PLATE 1

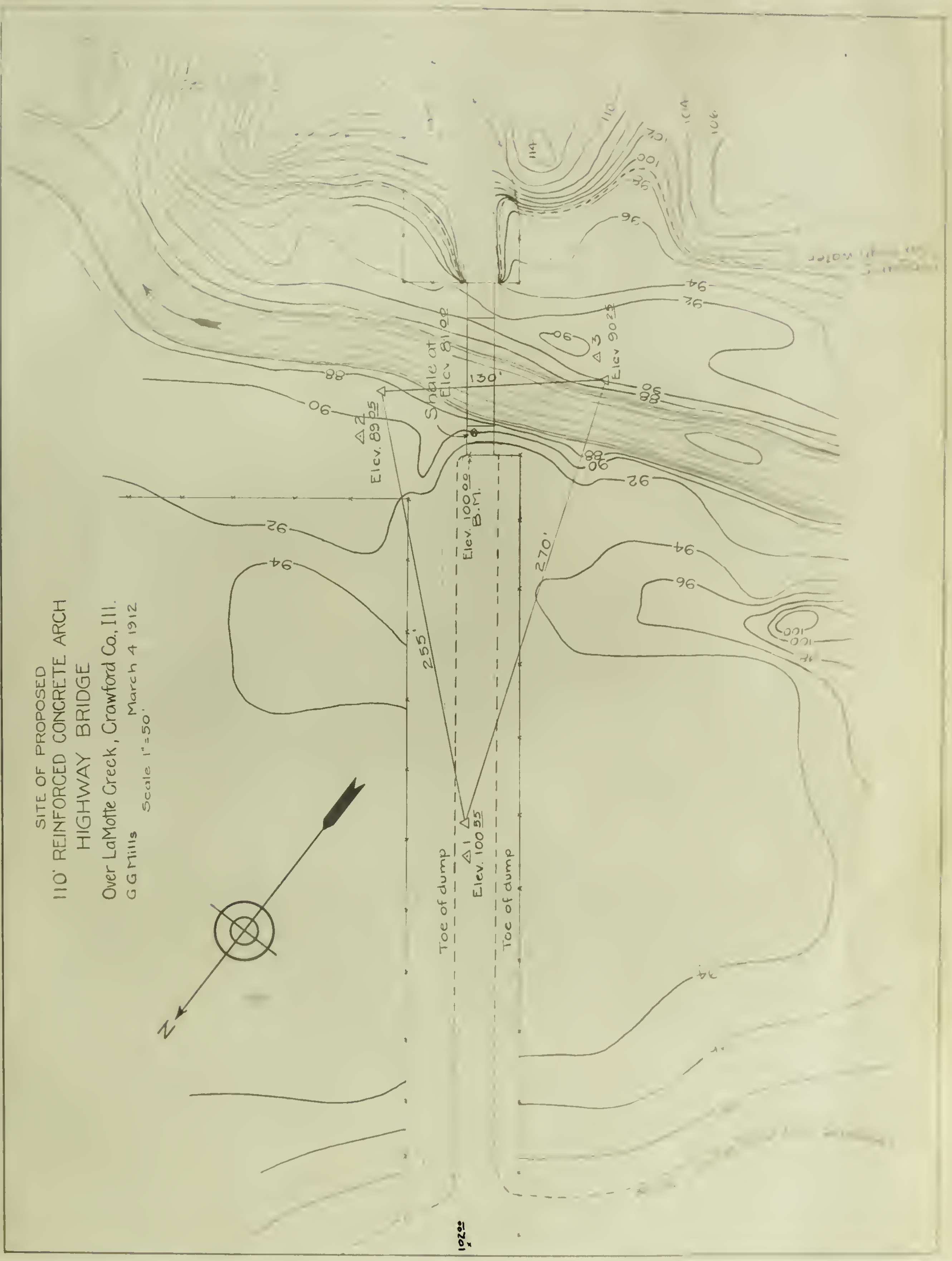

SITE OF PROPOSED
110' REINFORCED CONCRETE ARCH
HIGHWAY BRIDGE
Over LaMotte Creek, Crawford Co., Ill.
G G Mills
March 4 1912
Scale 1"=50'
Toe of dump
Toe of dump
Elev. 100.55
Elev. 100.00
B.M.
Shale at
Elev. 81.00
Elev. 89.55
Elev. 90.25
255'
270'
130'
N

Plate 1.

The soil is for the most part a sandy silt deposit which does not offer a very good foundation. As indicated on the map, however, this silt deposit is underlain by a heavy shale which outcrops in the creek bed two or three hundred feet down-stream from the bridge. This shale is about two feet in depth and is of good heavy quality. It is underlain by about six inches of coal which is followed by a heavy deposit of soapstone. A boring made under the bridge as shown, indicates the elevation of the shale at the west abutment. This elevation will be assumed to be the same for both sides of the stream.

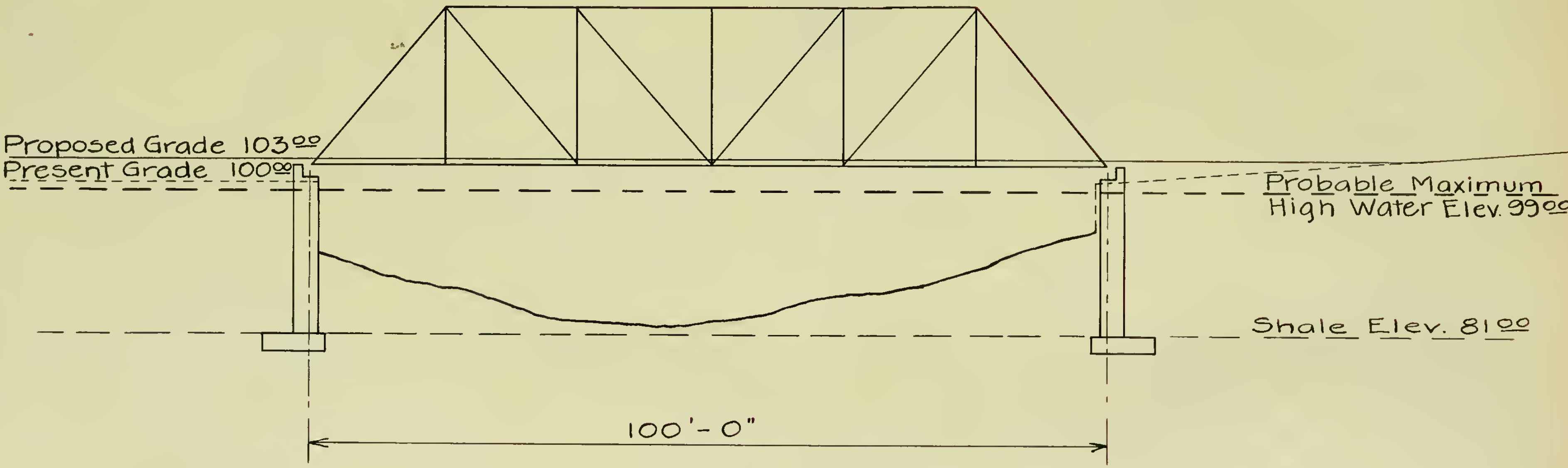
The four designs here discussed will be designated as Schemes A, B, C, and D.

SCHEME A

Scheme A is a 100' through Pratt truss with reinforced concrete floors. No actual design for this bridge was attempted, the weights of steel superstructure being taken from Prof.Dufour's "Highway Bridges". The floor was assumed to be of uniform six-inch thickness and no attempt made at specific design.

Since the "low steel" in the present bridge is submerged in time of very high water it was deemed advisable to raise the grade of the roadway so as to permit sufficient clearance to allow brush, logs, etc. to pass under the proposed bridge. The foundation material is of such poor quality that all foundations will be carried down to shale. The heights of abutments and piers will then depend upon the grade of the roadway.

# SCHEME A
## Through Truss
## Reinforced Concrete Abutments




| | Quantity | Unit Costs | Item. Cost | Grand Total | |
|---|---|---|---|---|---|
| | | | | Plain Abut. | Reinf. Abut. |
| Steel in trusses, beams, etc. | 64 220 # | $ 0.04 | $ 2568.80 | | |
| Concrete | | | | | |
| Floor | 29.6 cu. yd | 9.00 | 266.40 | | |
| Plain Abutments | 307.4 cu. yd. | 7.00 | 2151.80 | | |
| Reinf. Abutments | 125.9 cu. yd. | 12.00 | 1510.80 | | |
| Earthwork | | | | | |
| Excavation | 496.0 cu. yd. | 0.50 | 248.00 | | |
| Fill | 1110.0 cu. yd. | 0.25 | 277.50 | $ 5512.50 | $ 4971.50 |

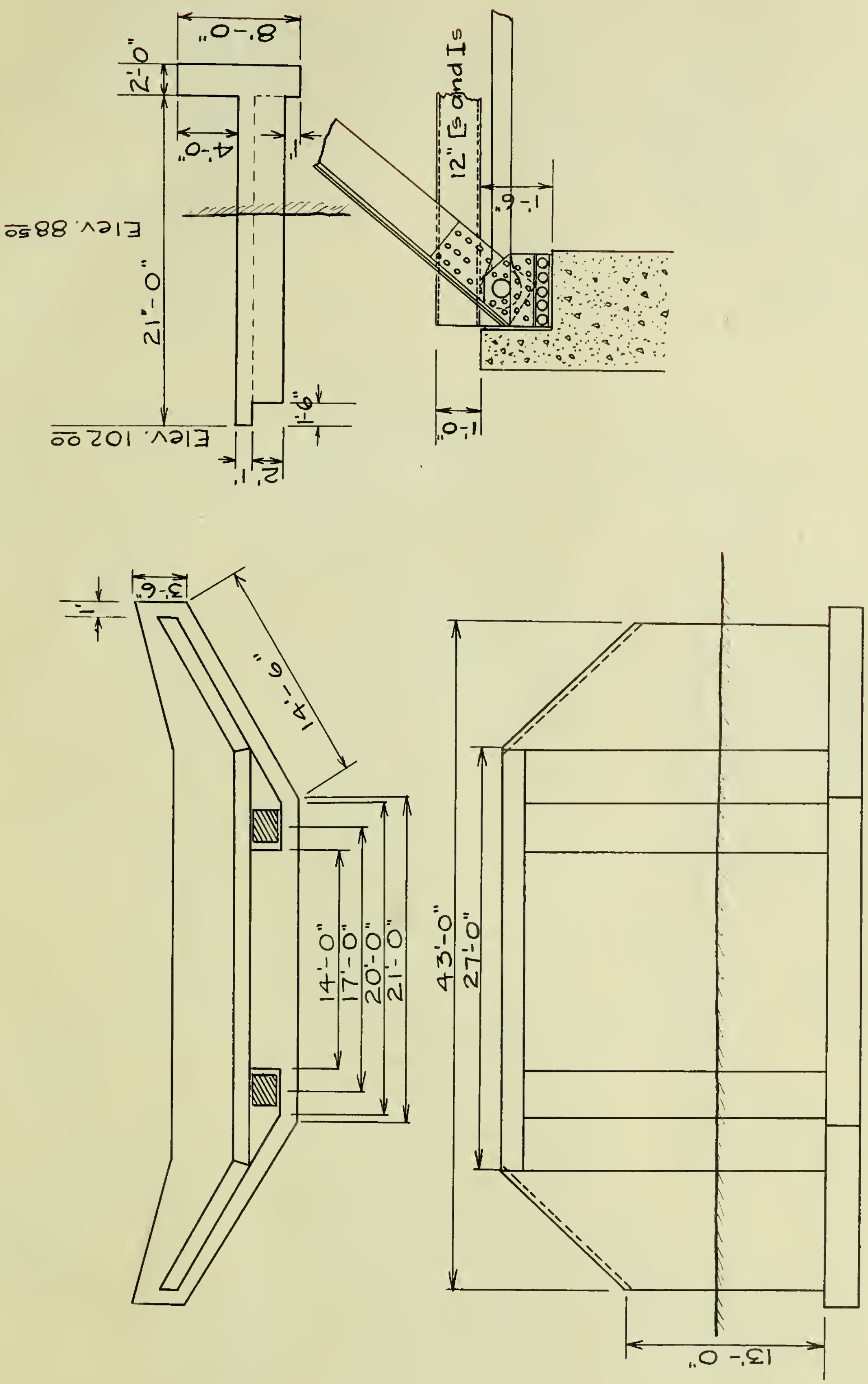

8'-0"
2'-0"
4'-0"
1'-1"
Elev. 88.50
21'-0"
Elev. 102.00
1'-6"
1'-1"
2'
12" [s and Is
1'-6"
1'-0"
3'-6"
1'-1"
14'-6"
14'-0"
17'-0"
20'-0"
21'-0"
43'-0"
27'-0"
13'-0"

Plate 3 gives the design for a reinforced abutment for the bridge of Scheme A, which is shown in Plate 2. For purposes of comparison, in the table of Plate 2 are given the costs of Scheme A with either plain or reinforced abutments. In this estimate the quantities for the plain concrete abutment are based upon the design of Plate 5 which was made for the pony truss bridge of Scheme B.

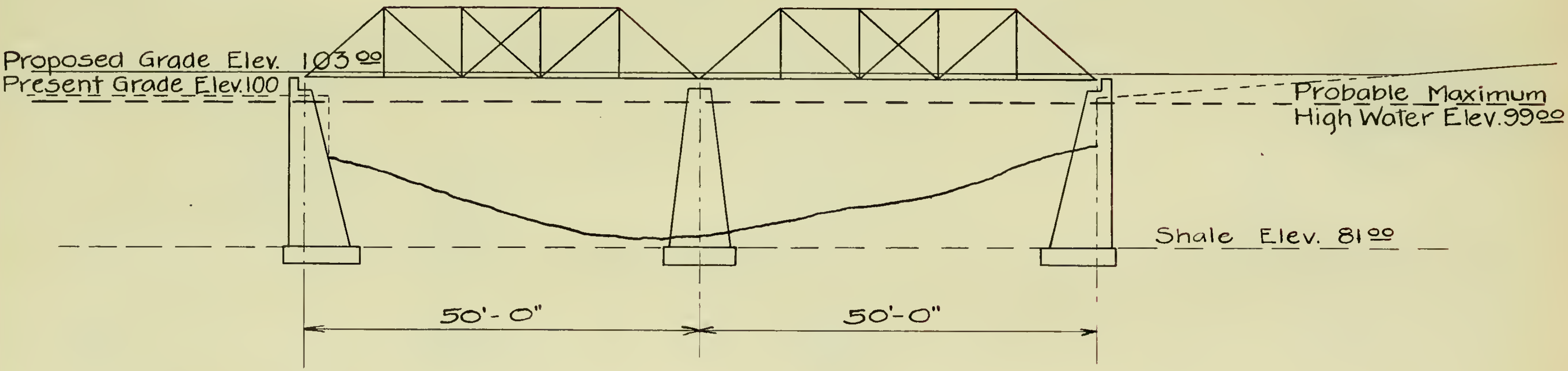
## SCHEME B

Scheme B is very similar to Scheme A in that the superstructure was not completely designed but that the quantities for same were taken directly from Prof. Dufour's "Highway Bridges". This bridge is also assumed to have concrete floors and to be 100 feet long (two spans of 50 feet each). The abutments and pier were designed in plain concrete but the table of Plate 4 gives comparative costs based upon the concrete abutment design of Plate 3 and the pier design of Scheme C.

Since the types of bridge represented by Schemes A and B have become standard and costs based upon such investigations as those made by Prof. Dufour are very accurate it was deemed unnecessary to go into the designs further than has been indicated above.

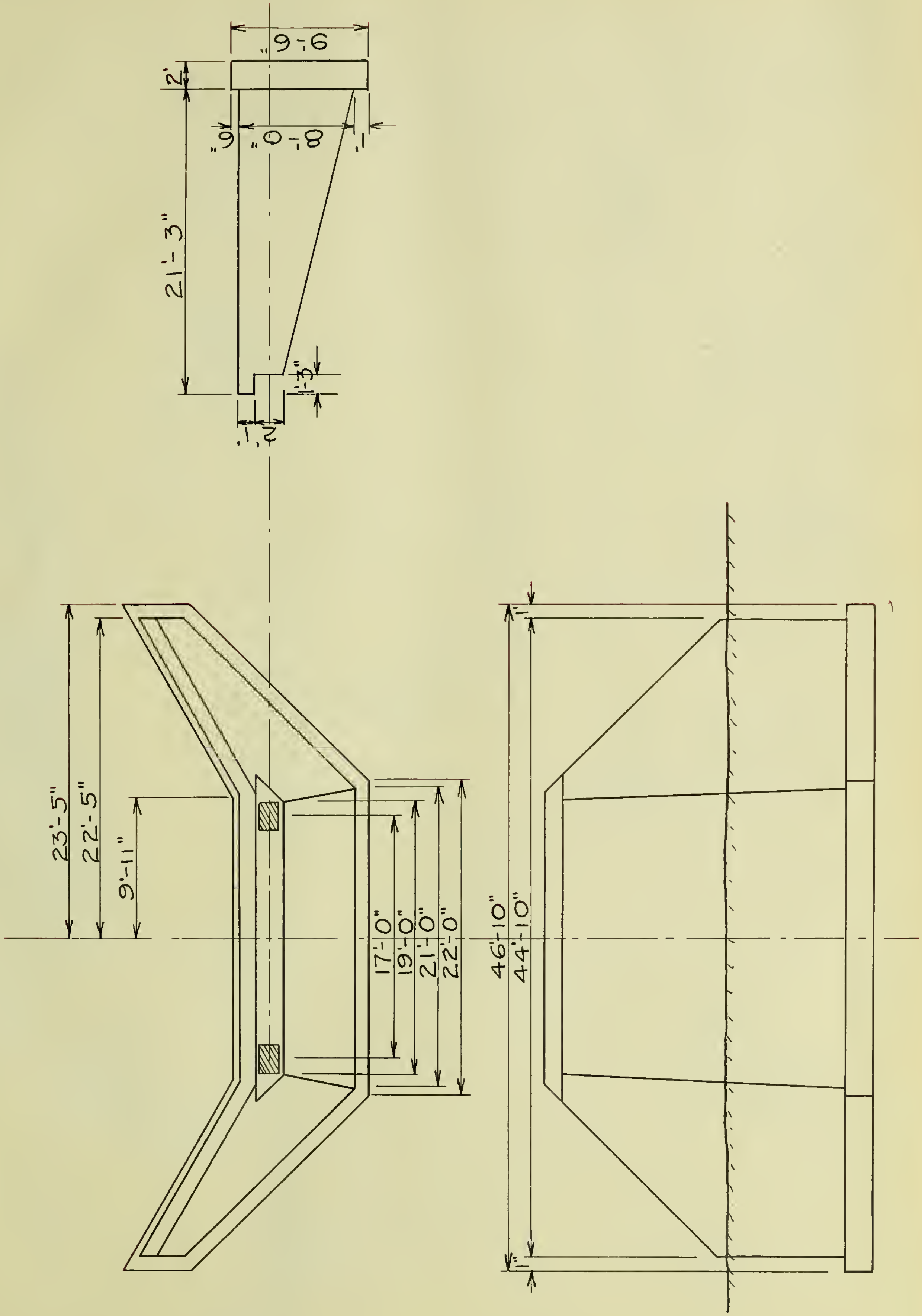
The types of bridge represented by Schemes C and D are, for various reasons, coming into more common use. Their design, however, has not as yet become standardized to so great an extent as have the designs in steel. Accordingly they will be more carefully discussed than have either of the preceeding schemes.

SCHEME B
2 Pony Truss Spans
Gravity Abutments and Pier



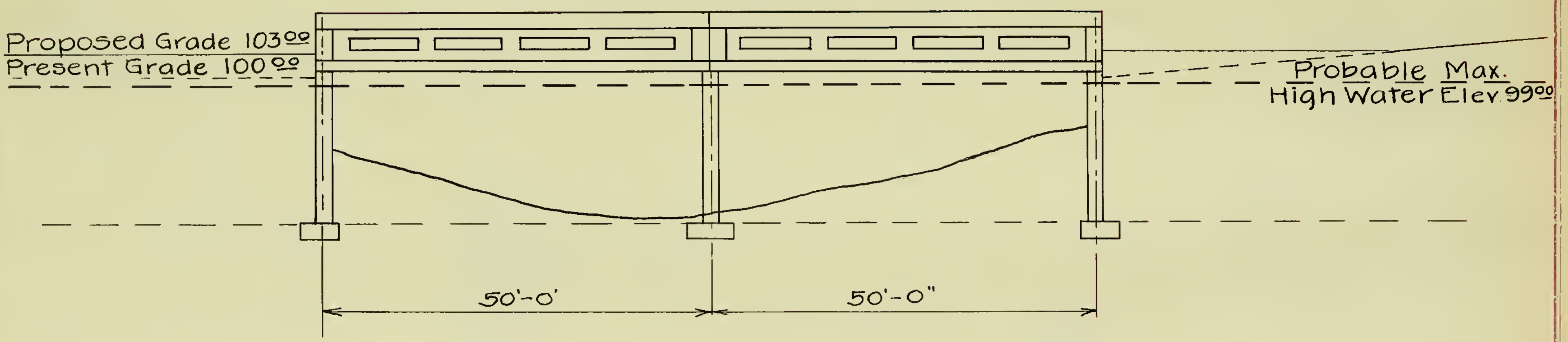

| | Quantity | Unit Costs | Item. Costs | Grand Total Plain Abut. | Grand Total Reinf. Abut. |
|---|---|---|---|---|---|
| Steel in trusses, etc. | 49 600 # | $ 0.04 | $ 1984.00 | | |
| Concrete: Floor | 29.6 cu.yd | 9.00 | 266.40 | | |
| Plain Abuts. | 307.4 cu.yd. | 7.00 | 2151.80 | | |
| Reinf. Abuts. | 125.9 cu.yd. | 12.00 | 1510.80 | | |
| Plain Pier | 111.8 cu.yd. | 7.00 | 782.60 | | |
| Reinf. Pier | 45.9 cu.yd. | 12.00 | 550.80 | | |
| Excavation: Plain Abuts. | 718.0 cu.yd. | 0.50 | 359.00 | | |
| Reinf Abuts. | 550.0 cu.yd. | 0.50 | 275.00 | | |
| Fill | 1110.0 cu.yd. | 0.25 | 277.50 | $ 5821.30 | $ 4864.50 |

PLATE 5

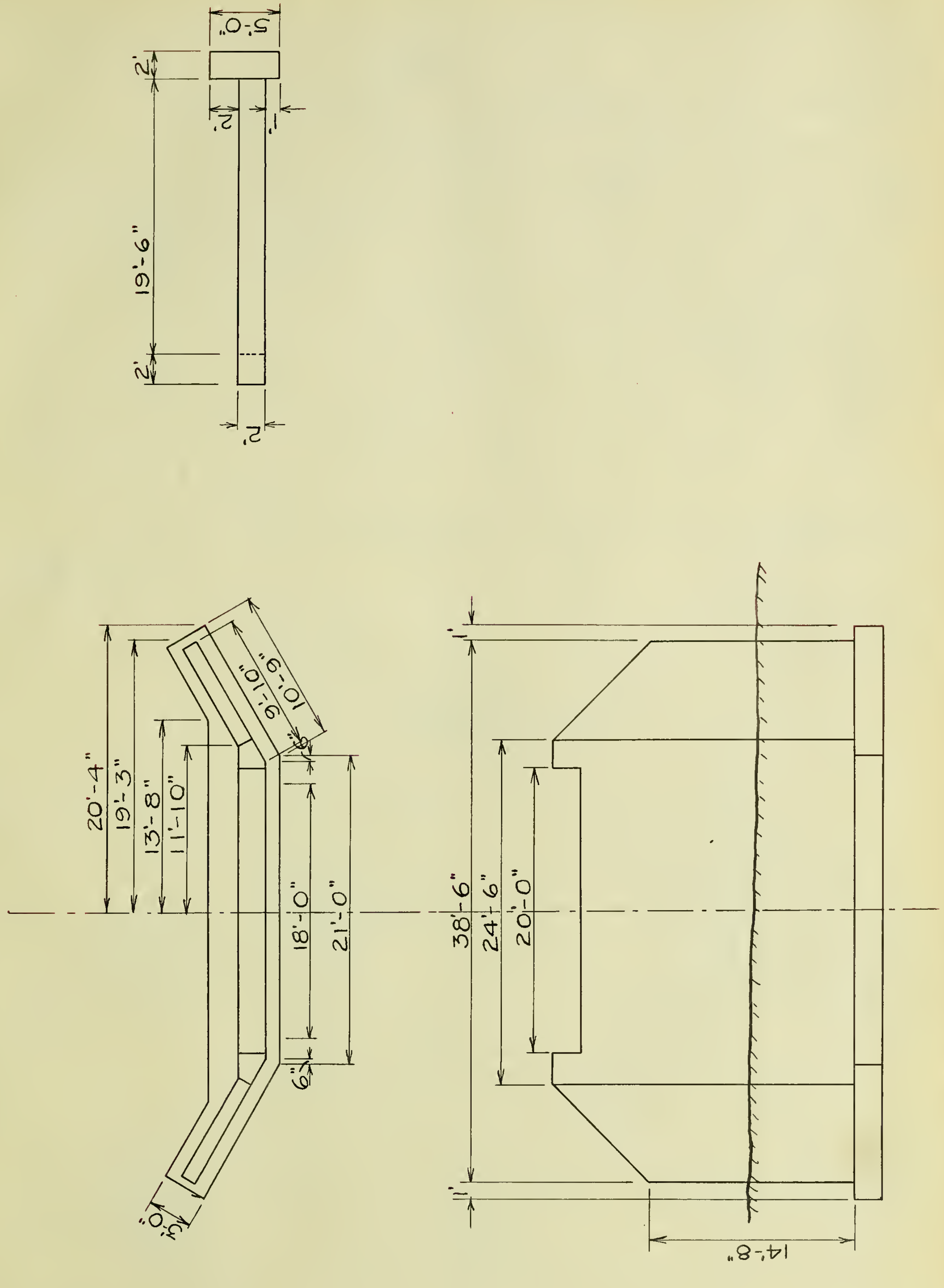


SCHEME C

2 - Concrete Girder Spans




| | Quantity cu.yds. | Unit Costs | Item. Costs | Grand Total |
|---|---|---|---|---|
| Concrete: | | | | |
| Floor System | 72.3 | $ 12.00 | $ 867.60 | |
| Girders | 90.8 | 12.00 | 1089.60 | |
| Abutments | 142.6 | 12.00 | 1711.20 | |
| Pier | 45.9 | 12.00 | 550.80 | |
| Excavation | 550.0 | 0.50 | 275.00 | |
| Fill | 1110.0 | 0.25 | 277.50 | $ 4771.70 |


5'-0"
2'
2'
1'
19'-6"
2'
2'
20'-4"
19'-3"
13'-8"
11'-10"
9'-10"
10'-6"
6"
18'-0"
21'-0"
6"
38'-6"
24'-6"
20'-0"
1'
1'
14'-8"

## SCHEME C

Scheme C consists of two reinforced concrete girder spans of 50 feet each, supported on reinforced concrete abutments and pier. The designs for the abutment and pier will not be discussed in detail, but will be taken as shown in Plate 7. An investigation of this design shows that the abutments are safe against overturning or sliding and that they have sufficient bearing on the shale. No effort will be made to design the reinforcement for the abutments.

### Loading

The loading for this bridge will be assumed as follows:-

Dead Load

Weight of concrete 140 lb. per cu. ft.

Weight of earth 100 lb. per cu. ft.

Live Load

Uniform load of 125 lb. per sq. ft.

Concentrated load due to 24 tons on two axles 10-feet apart--16 tons on rear axle and 8 tons on front. The loads will be taken as distributed over a width of 12 feet and in the slab design, over a length of 5 feet.

The moments will be computed for both uniform and engine live loads and the greater of the two combined with the dead load moment will be used in designing the girders and floor system.

Computations.

Dead Load Moments

Assume b = 24 inches

and d = 6 feet 6 inches

Girder = 2 x 6.5 x 140 = 1820 lb. per ft.

Floor = $\frac{1.0 \times 16.0 \times 140}{2}$ = 1120 lb. per ft.

Road Surface = $\frac{3/4 \times 16 \times 100}{2}$ = 600 lb. per ft.

Total 3640 lb. per ft.

$M = 1/8\ wl^2$

= 13650000 lb. inches

Live Load Moments

Engine loading

The loading shown in Figure 1 will give a maximum bending moment at the section under the 32000-lb. load

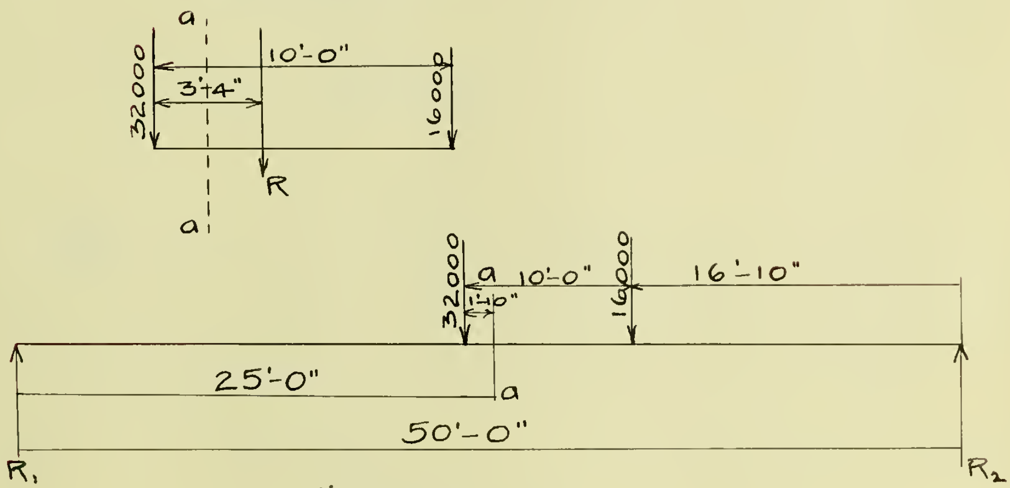



Figure 1.

By taking moments about $R_2$, $R_1$ is found to be 22580 lb.

$M = \frac{22580 \times (25.0-1.83)}{2}$

= 6260000 lb. inches

Uniform Load

125 lb. per sq. ft. over 12 ft. of width

$$M = \frac{1/8 \times 12 \times 125 \times 2500}{2}$$

$$= 2810000 \text{ lb. inches}$$

Combining the dead and engine load moments the value is given as

$$M = 13650000 + 3130000$$

$$= 16780000 \text{ lb. inches}$$

The unit compressive stress in concrete will be taken as 750 lb. and the unit tensile stress in the steel as 16000 lb.

From Plate 3 of Turneaure and Maurer's "Reinforced Concrete Construction" this gives

$$R = \frac{M}{bd^2}$$

$$= 136$$

$$d = \sqrt{\frac{M}{136b}}$$

$$= \frac{16780000}{136 \times 24}$$

$$= 72 \text{ inches}$$

Since the reinforcement will be placed in three rows add 6 inches to the value of d for the value of the total depth t.

Moment Reinforcement in Girder

$$A = pbd$$

$$= .01 \times 24 \times 72$$

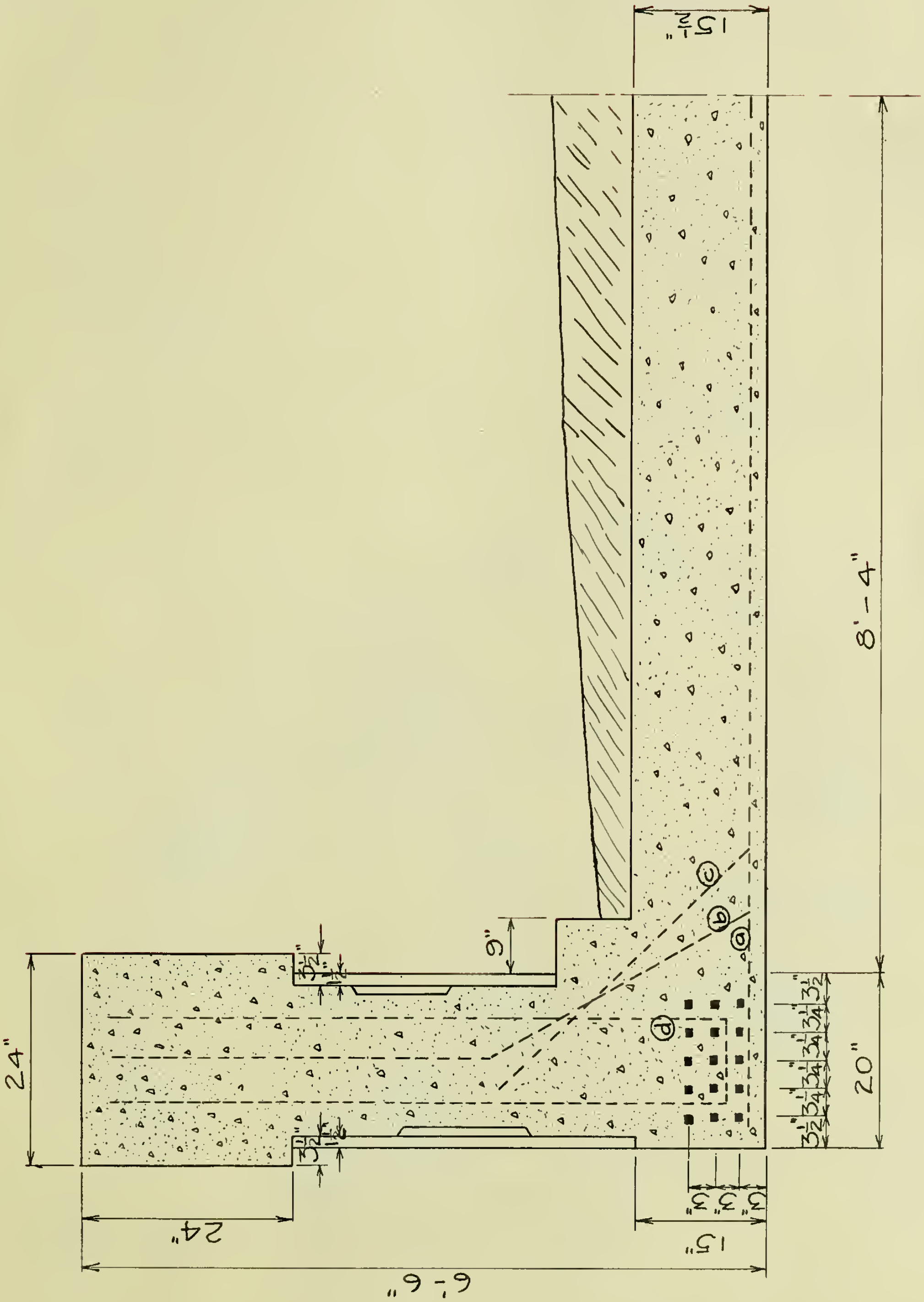
$$= 17.30$$

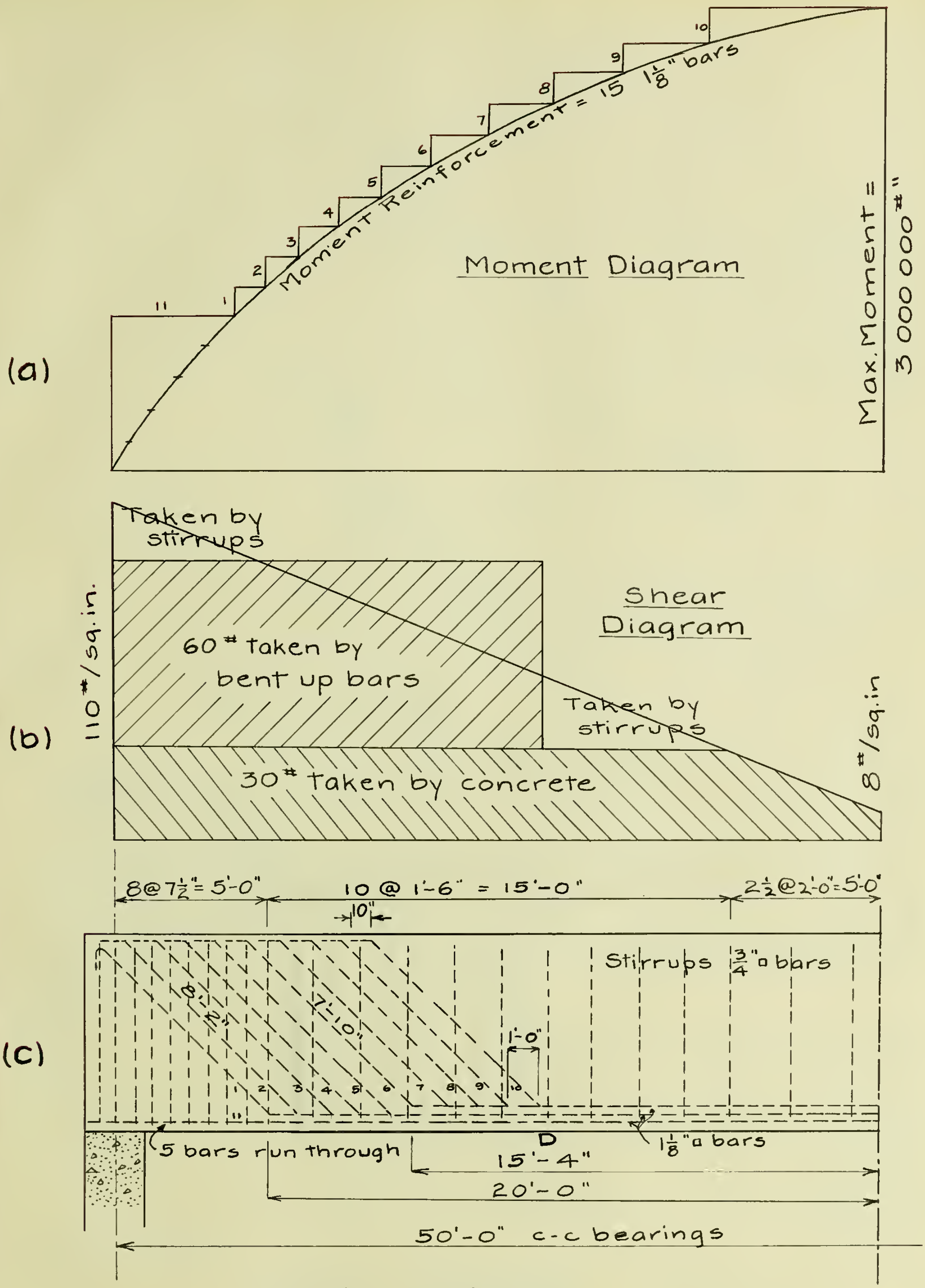
Use 1 1/8 in. square bars

$$a = 1.27 \text{ sq. in.}$$

$$\frac{17.30}{1.27} = 13.6$$

PLATE 8






Elevation of Girder
Showing distribution of steel

Use 15 1 1/8 in. sq. bars

Let U be the total bond stress.

$$U = \frac{V}{jd}$$

$$= \frac{128000}{7/8 \times 78}$$

$$= 1880 \text{ lb.}$$

Assume allowable unit bond stress = 80 lb.

$$\frac{1880}{80} = 23.5 \text{ inches}$$

$$\frac{23.5}{4 \times 1\ 1/8} = 5.2$$

Let 5 bars run through to provide for bond stress to take up the longitudional tension at ends of girders.

Web Reinforcement

Plate 9 illustrates the method for the design of web reinforcement. Figure (a) of Plate 9 gives a moment diagram for half of the girder upon which has been plotted the areas of the moment rods. The numbers on this diagram refer to the diagram (c) which is an elevation showing the method of bending up the rods and the dimensions of same.

Let the rods be turned up at angles of 45 degrees and spaced as indicated in (a), allowing five to run through as indicated above. The stress in these bars will be assumed at 12000 lb. per sq. in. The shear taken by the bent rods can now be computed.

$$S = \frac{A \times 12000}{12 \times 1.41 \times 15} = 60 \text{ lb.}$$

Figure (b) is the unit-shear diagram for the half girder of (a). Thirty pounds of the shear can be assumed to be carried by the

concrete. This is represented by the lower shaded portion of Figure (b). An additional 60 lb. can be carried by the rods where they are bent up. This is shown in the upper shaded portion of Figure (b). The only shear remaining unprovided for is represented by the unshaded portion under the shear curve. Theoretically the spacing of 3/4 inch stirrups to take up this shear (which is 20 lb. per sq. in.)is given as follows:-

$$\text{Total shear} = 20 \times 15 \times 72$$

$$= 21600 \text{ lb.}$$

$$S = \frac{A \times d \times 12000}{21600}$$

$$= \frac{1.125 \times 72 \times 12000}{21600}$$

$$= 45 \text{ inches}$$

Since stirrups are comparatively cheap they will be spaced at the ends to carry the total shear at the section. This shear at the end is 128000 lb. and at D is 50000 lb. From the above formula for S the spacing at the end is found to be 7 1/2 inches and that at D to be 18 inches. We will therefore space the stirrups at 7 1/2 inches for the first five feet of the girder, 18 inches for the next 15 feet and 24 inches in the middle five feet of the span.

## Floor System

### Computation of Moments

#### Engine load

Consider load of 16 tons uniformly distributed over 12 feet of width and 5 feet of length of bridge.

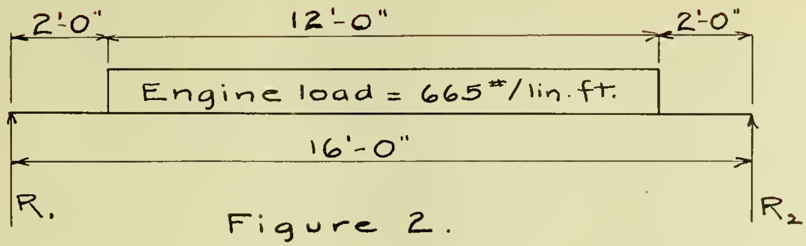



Figure 2.

$R_1 = 4000$ lb.

Moment at center

$M = (4000 \times 8 - 665 \times 6 \times 3) \times 12$

$= 240000$ lb. inches

Dead Load

Assume slab 1 ft. 3 in. thick

$M = 1/8 \times 175 \times 16^2 \times 12$

$= 67000$ lb. inches

Total moment

$M = 240000 + 67000$

$= 307000$ lb. inches

$$d = \sqrt{\frac{M}{136b}}$$

$$= \sqrt{\frac{307000}{136 \times 12}}$$

$= 13.6$ inches

Use $t = 15\ 1/2$ inches

Floor Reinforcement

$A = pbd$

$= .01 \times 12 \times 15.5$

$= 1.86$

Use 3/4 inch bars $a = .563$ sq. in.

$\frac{.563 \times 12}{1.86} = 3.64$ in. spacing

Space rods 5 1/2 inches

$$U = \frac{V}{jd}$$

$$= \frac{4000}{7/8 \times 15.5}$$

$$= 294 \text{ lb.}$$

$$\frac{294}{80} = 3.68$$

$$\frac{3.68}{3} = 1.23 \text{ rods per ft.}$$

Run every third rod through.

<u>Provision Against Failure by Tension in Web.</u>

Shear = 4000 lb. per lin. ft. of floor

Bend up every 3rd bar into web of girder to take this load. (See Plate 8)

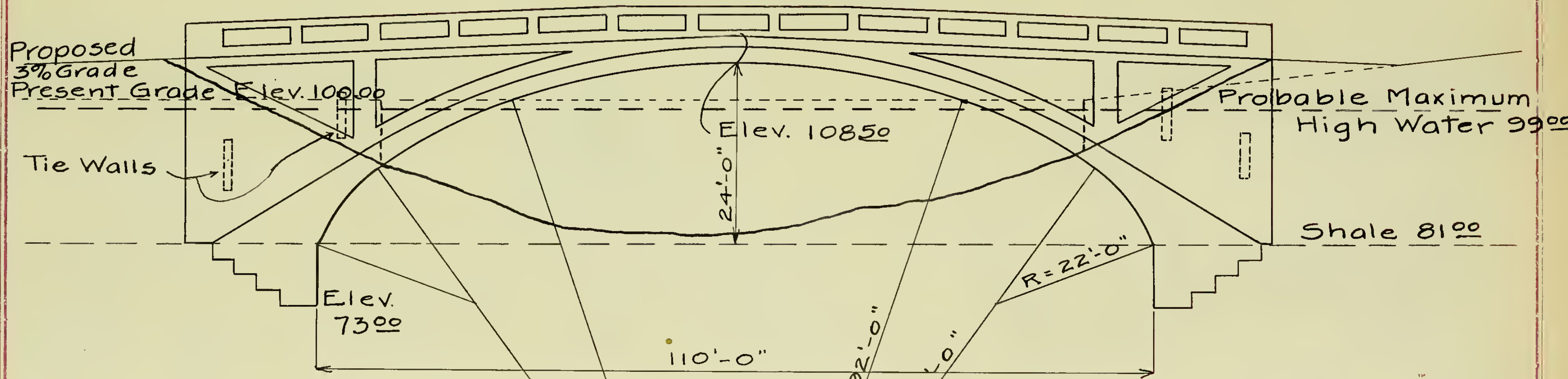
$$\text{Unit Stress in bar} = \frac{4000}{.563}$$

$$= 7100 \text{ lb.}$$

Below is bill of steel to be used in the bridge of Scheme C, exclusive of abutments.

| No. | Section | Length | No. Req. | Weight | Cost per lb. | Total Cost |
|---|---|---|---|---|---|---|
| Floor System (See plate 8) | | | | | | |
| a | $\frac{3}{4}$"□ | 19'-6" | 113 | | | |
| b | " | 21'-11" | 113 | | | |
| c | " | 17'-8" | 113 | 12770# | $2\frac{3}{4}$ ¢ | $351.20 |
| Girder | | | | | | |
| d | $\frac{3}{4}$"□ | 12'-6" | 200 | 10760# | $2\frac{3}{4}$ ¢ | |
| 1 | $1\frac{1}{8}$"□ | 58'-0" | 4 | | | |
| 2 | " | 56'-0" | 4 | | | |
| 3 | " | 54'-0" | 4 | | | |
| 4 | " | 52'-0" | 4 | | | |
| 5 | " | 50'-0" | 4 | | | |
| 6 | " | 48'-2" | 4 | | | |
| 7 | " | 46'-2" | 4 | | | |
| 8 | " | 44'-2" | 4 | | | |
| 9 | " | 42'-2" | 4 | | | |
| 10 | " | 40'-2" | 4 | 8440# | $2\frac{3}{4}$ ¢ | |
| 11 | " | 51'-6" | 20 | 4430# | $2\frac{3}{4}$ ¢ | $649.80 |
| | | | | Total | | $1001.00 |

# SCHEME D
## 110'- Reinforced Concrete Arch




| | Quantities cu.yd. | Unit Costs | Item. Costs | Grand Total |
|---|---|---|---|---|
| Concrete: Arch | 205.0 | $15.00 | $3075.00 | |
| Abutments | 237.0 | 12.00 | 2844.00 | |
| Span.Walls | 194.0 | 12.00 | 2328.00 | |
| Tie walls | 6.0 | 12.00 | 72.00 | |
| Earthwork: | | | | |
| Excavation | 842.0 | 0.50 | 421.00 | |
| Fill: Spandrel | 840.0 | 0.25 | 210.00 | |
| Grades | 1570.0 | 0.25 | 392.50 | $9342.50 |

## SCHEME D.

Scheme D consists of a design for a concrete arch bridge for the site discussed above. In order to provide adequate waterway it is found necessary to use a clear span of 110 feet between abutments and a rise of 24 feet. The curve of the arch ring may be found approximately by the method of "two circles" given below.

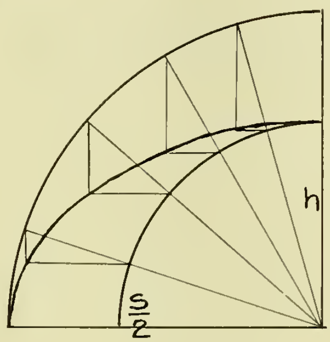
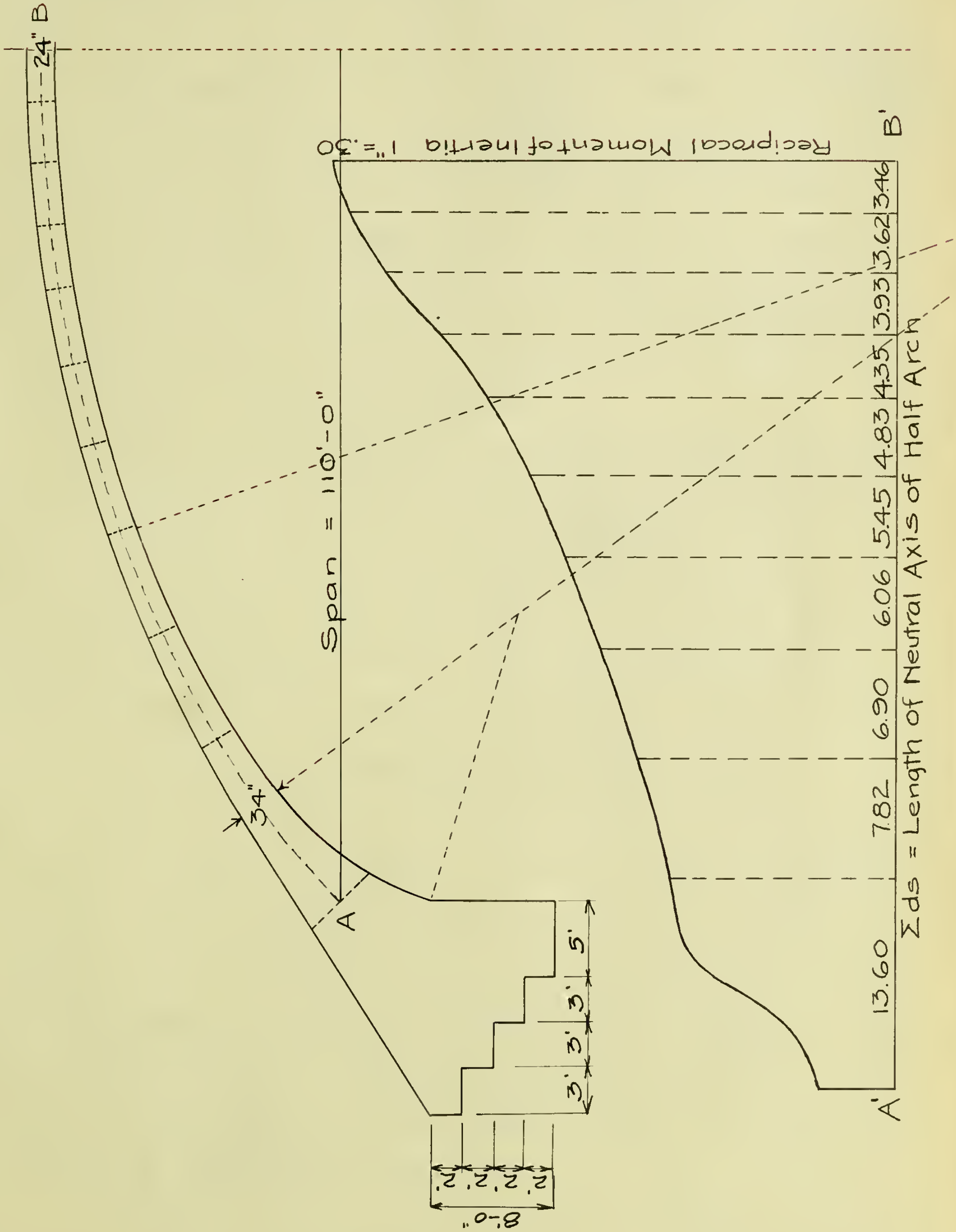



Figure 3

In the figure above let h be the rise and S the span of a given arch. About O as a center strike arcs of radii h and $\frac{S}{2}$. Then at various points produce lines from center cutting both arcs. Draw horizontal lines through the intersections of these radial lines with the inside circle, and vertical lines through their points of intersection with the outside circle. The intersection of corresponding lines drawn in this way will lie on the approximate curve desired. It will be more convenient to make the arch three or five centered, so after having secured the approximate curve by this method it is easily determined by trial the radii best to use in order to secure practically the same curve, using several centers. For the arch under discussion five centers will be used, the lengths of radii being given in Plate 10.

The thickness of the arch ring will be taken as 2 feet at the crown and 3 feet at points 50-feet from the center. From


24" B
Span = 110'-0"
Reciprocal Moment of Inertia 1" = .30
B'
A
34"
A'
13.60
7.82
6.90
6.06
5.45
4.83
4.35
3.93
3.62
3.46
Σds = Length of Neutral Axis of Half Arch
5'
3'
3'
3'
2'
2'
2'
2'
8'-0"

these latter points the extrados will be continued as straight lines, intersecting the horizontal line through the bottom of the arch at points 14 feet back of same. The abutment footing will be carried down into the shale 8 feet, by steps as shown in Plate 11.

The construction of the bridge now under discussion will necessitate raising the elevation of the road over the crown of the arch to 108.50 or 8 1/2 feet above its present grade. It is proposed that the road on either side be given a 3 per cent grade from the top of the bridge to the point where it intersects the present grade. The quantities for fill will take this change of grade into account.

Having assumed dimensions for the curve and thickness of the arch ring and abutments it will next be necessary to investigate the arch thus assumed to find whether it is a safe and economical design. This investigation will now be made according to the method outlined in Turneaure and Maurer's "Principles of Reinforced Concrete Construction", Chapter VIII.

<u>Division of Arch Ring to make $\frac{ds}{I}$ Constant.</u>

Lay out the arch ring to scale as shown in Plate 11. The section A-B will be considered to act in accordance with the arch theory, the portion below A being considered as abutment. Lay off the line $A^1B^1$ equal in length to the neutral axis AB. Now, at the tenth points along the curve compute the Moments of Inertia of the sections, assuming 3/4 inch square bars spaced 4 inches top and bottom. In the following table are tabulated the results of these computations.

## Division of Arch Ring

| No. of Division | Depth $d$ | $I_c$ | $15I_s$ | $I = I_c + 15I_s$ | $i = \frac{1}{I}$ | $\delta s$ |
|---|---|---|---|---|---|---|
| 1 | 2.04 | 0.707 | .23 | 0.93 | 1.06 | 3.46 |
| 2 | 2.09 | 0.760 | .25 | 1.01 | .95 | 3.62 |
| 3 | 2.21 | 0.900 | .27 | 1.17 | .77 | 3.93 |
| 4 | 2.29 | 1.005 | .30 | 1.30 | .67 | 4.35 |
| 5 | 2.37 | 1.110 | .34 | 1.45 | .63 | 4.83 |
| 6 | 2.46 | 1.240 | .37 | 1.61 | .56 | 5.45 |
| 7 | 2.54 | 1.365 | .40 | 1.77 | .51 | 6.06 |
| 8 | 2.62 | 1.500 | .43 | 1.93 | .45 | 6.90 |
| 9 | 2.71 | 1.660 | .46 | 2.12 | .42 | 7.82 |
| 10 | 2.79 | 1.812 | .50 | 2.31 | .20 | 13.60 |
| | | | | | 6.22 | |

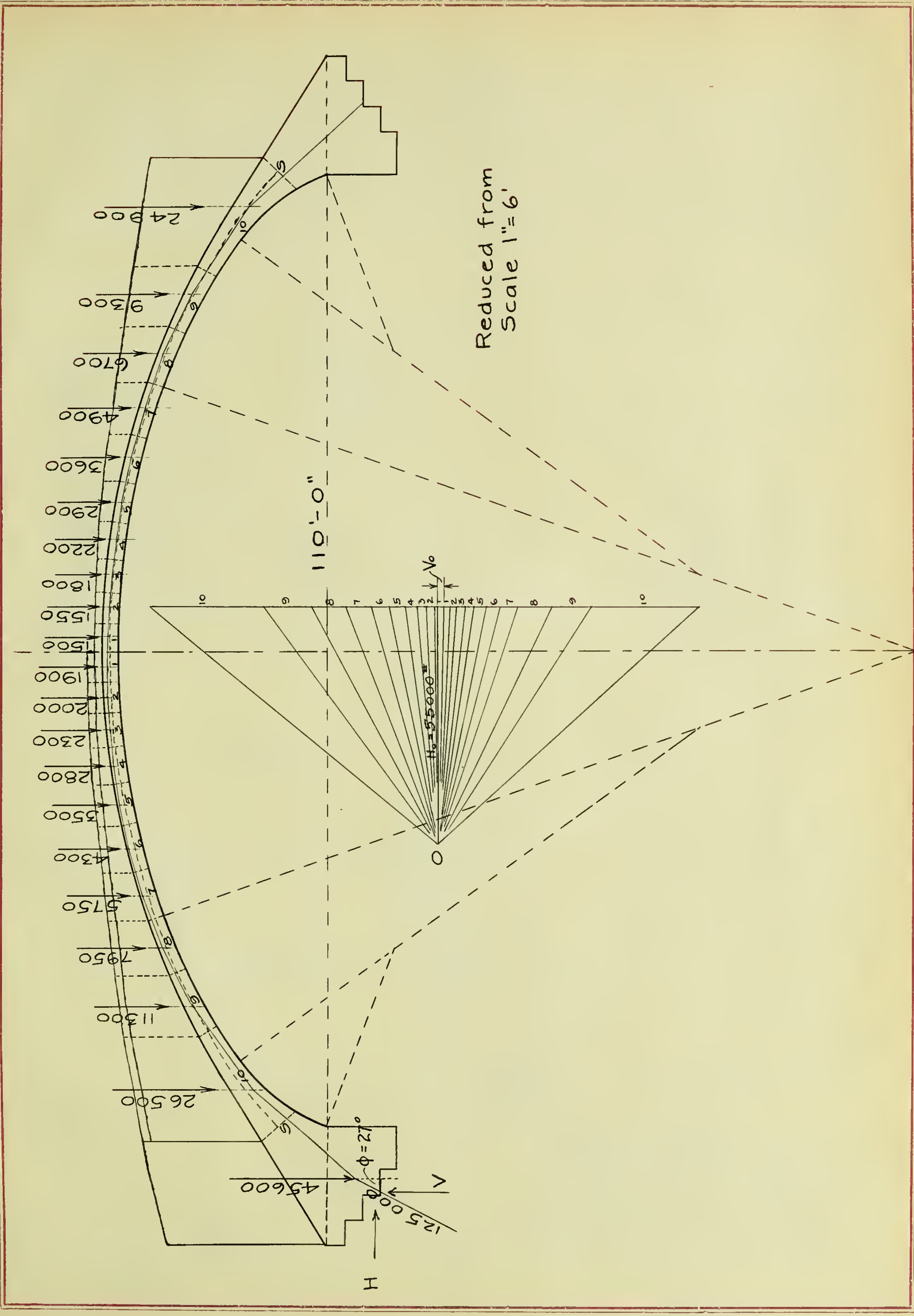
$$\frac{\delta s}{I} = \frac{60.0 \times .622}{10}$$

$$= 3.73$$

From these values of the Moment of Inertia their reciprocals are computed and placed in column 6 of the table.

At the tenth points on the line $A^1B^1$ plot points representing the reciprocals of the Moments of Inertia. Draw a curve through the points thus plotted. It is now necessary to divide the line $A^1B^1$ into such divisions that these lengths multiplied by their corresponding ordinates to the curve will be constant. This is done by trial.

The values for ds once obtained it will be necessary, if the graphical method of analysis is used, to lay out the arch with its reduced load contour to a large scale. As in the preceeding schemes the weight of concrete will be taken


Reduced from
Scale 1"=6'
110'-0"
$H_0=35000^{\#}$
$V_0$
O
24900
9300
6700
4900
3600
2900
2200
1800
1550
1500
1900
2000
2300
2800
3500
4300
5750
7950
11300
26500
45600
125000
$\phi=27°$
H
V

at 140 lbs. per cu. ft. and that of earth at 100 lb. per cu. ft. The loading considered will be that of a live load over one half the bridge, since this loading gives greatest displacement to the line of resistance. This live load will be assumed as 125 lb. per sq. ft.

The arch laid out and the points where $\frac{ds}{I}$ = Constant established, the reduced load contour is drawn as shown in Plate 12. Then the load coming upon each section is computed.

From the large drawing represented by Plate 12, the values of x and y given in columns 2 and 3 of Plate 13 are scaled. The values of m shown in columns 6 and 7 are computed by considering each half of the arch as a cantilever. From these values those given in columns 8 and 9 are readily computed.

$$H_o = \frac{n \Sigma my - \Sigma m \Sigma y}{2 (\Sigma y)^2 - n \Sigma y^2)}$$

$$= 55000 \text{ lb.}$$

$$V_o = \frac{\Sigma (m_R - m_L)x}{2 \Sigma x^2}$$

$$= 1270 \text{ lb.}$$

$$M_o = -\frac{\Sigma m + 2H_o \Sigma y}{2n}$$

$$= + 16400 \text{ lb.}$$

$$\text{Eccentricity at center} = \frac{M_o}{H_o}$$

$$= + .30 \text{ feet}$$

Having computed the values of $H_o$ and $V_o$ above, the columns 10 and 11 are readily filled.

The bending moment $M = m + M_o + H_o y \pm V_o x$. Having already computed the various quantities entering into this equation, the total bending moments tabulated under 12 and 13

# ARCH ANALYSIS

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|
| Points | x | y | $x^2$ | $y^2$ | $m_L$ | $m_R$ | $(m_L+m_R)y$ | $(m_R-m_L)x$ | $H_o y$ | $V_o x$ |
| 1 | 3.4 | 0.06 | 10.5 | .004 | 0 | 0 | 0 | 0 | 3 300 | 4 320 |
| 2 | 7.0 | 0.20 | 49.0 | .04 | − 10 500 | − 7 900 | − 3 700 | + 18 200 | 11 000 | 8 900 |
| 3 | 10.9 | 0.60 | 119.0 | .36 | − 28 900 | − 22 600 | − 30 900 | + 68 900 | 33 000 | 13 800 |
| 4 | 14.2 | 1.2 | 202.0 | 1.44 | − 53 600 | − 41 600 | − 114 000 | + 170 000 | 66 000 | 18 100 |
| 5 | 20.0 | 2.1 | 400.0 | 4.40 | − 108 700 | − 85 200 | − 406 000 | + 470 000 | 116 000 | 25 400 |
| 6 | 25.3 | 3.5 | 640.0 | 12.25 | − 183 200 | − 144 700 | − 1 150 000 | + 975 000 | 193 000 | 32 200 |
| 7 | 31.0 | 5.2 | 960.0 | 27.00 | − 282 800 | − 226 300 | − 2 640 000 | + 1 750 000 | 286 000 | 39 400 |
| 8 | 37.2 | 7.7 | 1380.0 | 59.00 | − 442 400 | − 357 300 | − 6 150 000 | + 3 160 000 | 424 000 | 47 300 |
| 9 | 44.0 | 11.7 | 1930.0 | 137.00 | − 673 900 | − 553 100 | − 14 450 000 | + 5 310 000 | 645 000 | 55 900 |
| 10 | 51.0 | 16.3 | 2600.0 | 266.00 | − 980 000 | − 815 200 | − 29 220 000 | + 9 080 000 | 900 000 | 64 800 |
| Σ | | 48.6 | 8290.5 | 507.49 | − 2 764 000 | − 2 253 900 | − 54 164 600 | + 21 002 600 | | |
| | | | | | − 5 017 900 | | | | | |
| Spring. | 55.0 | 20.0 | 3030.0 | 400.0 | 1 232 600 | 1 052 300 | 45 698 000 | 9 920 000 | 1 100 000 | 70 000 |

| | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Points | Bending Moment | | Thrusts | | Ecc. Dist. | | h ft. | $\frac{e}{h}$ | | % | $f_c$ | | $f_s$-tension | |
| | Left | Right | Left | Right | Left | Right | | Left | Right | Reinf. | Left | Right | Left | Right |
| 1 | 24 000 | 15 400 | 55 000 | 55 000 | +0.44 | +0.28 | 2.02 | .204 | .139 | 1.16 | 300 | 230 | | |
| 2 | 25 800 | 10 600 | 55 000 | 55 100 | +0.47 | +0.19 | 2.08 | .225 | .091 | 1.13 | 285 | 200 | | |
| 3 | 34 300 | 13 000 | 55 100 | 55 200 | +0.62 | +0.24 | 2.15 | .288 | .112 | 1.09 | 250 | 200 | | |
| 4 | 46 900 | 22 700 | 55 200 | 55 400 | +0.85 | +0.41 | 2.21 | .386 | .186 | 1.06 | 430 | 240 | | |
| 5 | 49 100 | 21 800 | 55 700 | 55 700 | +0.88 | +0.39 | 2.29 | .384 | .170 | 1.02 | 400 | 240 | | |
| 6 | 58 400 | 32 500 | 56 200 | 56 100 | +1.03 | +0.58 | 2.38 | .433 | .244 | 0.98 | 430 | 280 | 7100 | 3450 |
| 7 | 59 000 | 36 700 | 57 100 | 57 000 | +1.03 | +0.64 | 2.47 | .418 | .259 | 0.95 | 410 | 290 | | |
| 8 | 45 300 | 35 800 | 59 000 | 58 600 | +0.72 | +0.61 | 2.58 | .279 | .236 | 0.91 | 330 | 270 | | |
| 9 | 46 500 | 49 300 | 62 100 | 61 000 | +0.75 | +0.81 | 2.71 | .277 | .299 | 0.86 | 310 | 320 | | |
| 10 | 1 200 | 36 400 | 68 000 | 65 800 | +0.17 | +0.55 | 3.50 | .049 | .157 | 0.67 | 150 | 200 | | |
| Spring. | − 56 200 | − 5 900 | 86 400 | 81 800 | −0.65 | −0.07 | 5.33 | .122 | .013 | 0.44 | 165 | 95 | | |

are computed. The thrusts are scaled from the force polygon, and the eccentric distances are computed from the relation $e = \frac{M}{T}$ where M is the total moment at the section and T is the thrust. This assumes that the thrust is vertical to the section but the error arising from that assumption is small enough to be neglected. The values of $f_c$ are found to be smaller than the unit allowable stresses at all sections. Since the largest value occurs at the sixth point, that section will also be investigated for tensile stress in the steel. As shown in tables 25 and 26 this is only half the allowable tensile stress in steel so the reinforcement is more than sufficient.

Though the stresses are all within the maximum allowable, it is evident that the arch ring should be raised slightly so as to make the line of resistance approach more nearly the neutral axis. This can be accomplished by lengthening the radius of the central arc and shortening the radii of the side arcs slightly. Since these changes would not materially effect the quantities or stresses no recomputation will be made.

Investigation of Abutment Footings

To find the final thrust at the loaded end of the bridge the thrust at the last section of the arch is combined with the load due to weight of the abutment and fill over it. This gives a value of 125000 lb. for the thrust, making an angle of approximately 27° with the vertical. The values for H and V at the footing are now found to be 56000 lb. and 110000 lb. respectively.

The coefficient of friction for soapstone (which is

the material underlying the shale and into which the foundation penetrates) will be taken as .25. The amount of the horizontal component which may be assumed as taken up by the friction of the stone will then be .25 x 45600 = 11400 lb. This leaves 56000 - 11400 = 44600 lbs. to be taken up by the stone in direct horizontal compression. The unit horizontal pressure is therefore equal to $\frac{44600}{8}$ = 5575 lb. per sq. ft.

The vertical pressure will vary slightly from toe to heel of footing but will be approximately uniform. This pressure will be $\frac{110000}{14}$ = 7850 lb. per sq. ft.

For the actual material of the foundation these values are rather high but no redesign will be attempted. The thrust at the right end is considerably smaller than that at the left end so no investigation will be made of that footing.

Spandrel Walls

The spandrel walls will be designed to carry the pressure of the spandrel filling. For purposes of this estimate they will be assumed to be 12 inches thick at the top and 15 inches thick at the bottom. The hand rail will be 6 inches thick with 1 inch panel on either side-and 4 feet high.

In order that the spandrel walls not fail by falling outward, tie walls must be provided as shown in Plate 10. They will be made 9 inches thick and 3 feet deep and will be reinforced to take a tensile load equal to the pressure of the earth upon the wall-in addition to being designed as beams to support their own weight.

## Cost Data

An estimate will now be made of the costs of the various schemes above discussed. The results of this investigation are found in the following table- the itemized accounts being given on the plates containing the general layouts.

| Scheme | Cost |
|---|---|
| A - Plain Abuts. | $ 5512.50 |
| Reinf. Abuts. | 4971.50 |
| B - Plain Abuts. | 5821.30 |
| Reinf. Abuts. | 4864.50 |
| C | 4771.70 |
| D | 9342.50 |

From what has been discovered in the above it may be reasonably assumed that the concrete arch of Scheme D is not practicable for this particular site. Aside from the item of its prohibitive cost is the great uncertainty as to the action of the foundation material found here under such thrusts as would come upon it under the weight of the arch. The steel bridges are both practical and it would seem that reinforced abutments are to be preferred to those of plain concrete. Assuming all other considerations of equal weight, the concrete girder with reinforced abutments and pier would be preferable on account of its low initial cost. Aside from that, however, the author would recommend this form of structure on account of its permanency, and lack of annual maintainance expense. Where roads are well established, as most of the roads in this particular locality now are, the bridges and culverts should be constructed with a mind to more than the mere initial cost.


Monastery Hill Bindery